

Maysan NHS

**Proposed Shallow Foundation
Design for the Maysan National
High School – Annex Building**
Design Calculation

June 22, 2022

Prepared By

CEFOUND Group 1

This report takes into account the particular instructions and requirements of our client.

It is not intended for and should not be relied upon by any third party and no responsibility is undertaken to any third party.

De La Salle University – Manila
Civil Engineering Department**CEFOUND – Design Project****Document Verification**

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Section	CEFOUND - EH	
Prepared by	Name	Signature
	Abello, Giorgio A.	
	Canlas, Maureen Angelique A.	
	Ondevilla, Miguel Lorenzo A.	
	Samson, Jemina Mikaela T.	
	Villanueva, Daniel Kenneth	
Checked by	Dr. Lessandro E. Garciano	
Remarks		

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1 Introduction

1.1 Purpose of Design Calculation

Maysan National High School, located in the city of Valenzuela, plans to construct a new building, the Maysan National Highschool – Annex Building. This paper aims to propose shallow foundation designs for the said building according to the minimum design requirements of the National Structural Code of the Philippines (NSCP). Moreover, the proposed design is intended to be cost-effective and economical.

The objectives of the design project are as follows:

- Propose a shallow foundation design and calculation for the Maysan National Highschool – Annex Building
- Provide CAD drawings of the final foundation designs
- Tabulate the steel quantity and weight to calculate the estimated cost of the proposed foundation design

1.2 Project Location

The proposed design for shallow foundation is for the Annex building of Maysan National High School, located in Molave, Valenzuela City, Metro Manila. It is located approximately 12.6 kilometers northwest of the West Valley Fault System and is expected to experience ground shaking up to intensity VII. Ground shaking effects can be mitigated by following the provisions in the National Structural Building Code and Structural of the Philippines. It is safe from ground rupture, earthquake-induced landslides, liquefaction, and tsunami (Hazard Hunter PH, 2022).

2 Design Criteria

2.1 Units of Measurement

The standard unit of measurement used in this project is in Metric System. Accordingly, all length measurements are in millimeters (mm) and meters (m), forces in kilonewton (kN) and newton (N), bending moments in kilonewton-meter (kN-m), stresses in kilopascal (kPa) and megapascal (MPa).

2.2 Design Loads

2.2.1 Dead Loads

A. Superimposed Load

Roof Loads

Trusses	0.60 kPa (13 psf)
Purlins and Miscellaneous	0.15 kPa (3 psf)
Roofing Material	0.04 kPa (0.8 psf)
Insulation	0.04 kPa (0.8 psf)
Ceiling	0.24 kPa (5 psf)
Electrical Fixtures	0.10 kPa (2 psf)

Floor Load

25 mm Floor Finish	0.77 kPa (16 psf)
Partition	0.96 kPa (20 psf)
Electrical and Mechanical Fixtures	0.20 kPa (4.1 psf)
Ceiling System	0.20 kPa (4.2 psf)
Wall (4" Medium Density CHB both face plastered)	3.17 kPa (66 psf)
Stair	3.52 kPa (74 psf)

B. Unit Weight of Materials

Structural Steel	77.0 kN/m ³ (490 pcf)
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Reinforced Concrete	23.6 kN/m ³ (150 pcf)
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2.2.2 Live Loads

Classrooms	1.90 kPa (40 psf)
Corridors	3.80 kPa (80 psf)
Stairs	4.80 kPa (100 psf)
Roof	0.60 kPa (13 psf)

2.2.3 Lateral Loads

A. Earthquake Loads (Based on the parameters specified in the NSCP 2015)

Earthquake forces are generated by very dynamic events. For certain types of structures, a static equivalent method may be used to estimate the forces applied to the structure.

Design Base Shear	$V = \frac{C_v I}{R T} W$
Seismic Source Type	A
Seismic Zone	Z = 0.4
Importance Factor	I = 1.50
Soil Type	S _D
Numerical Coefficient	R = 8.5
Closest Distance to known Seismic Source	12.8 km
Near Source Factor	N _a = 1.00
	N _v = 1.1
Seismic Coefficient	C _a = 0.4
	C _v = 0.616
Structural Period	T = 0.4787 s

B. Wind Loads (Based on the parameters specified in the NSCP 2015)

The NSCP 7th edition specifies the procedures for generating the design winds loads for buildings and other structures. An outline of the overall process for the determination of wind loads as shown in Figure 2.2.3.1.

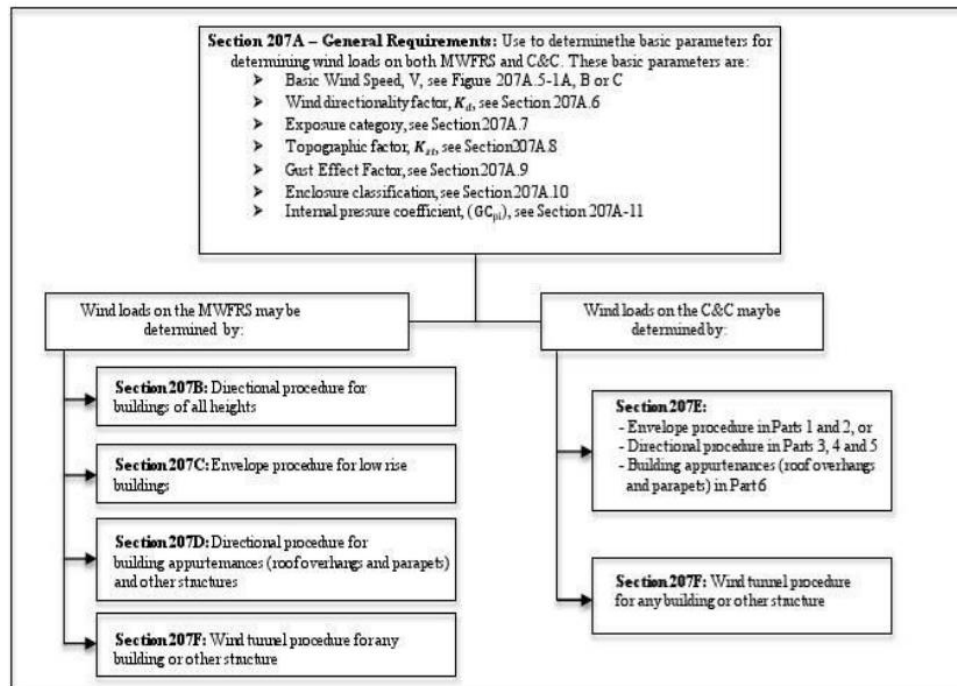


Figure 2.2.3.1. Process in determining wind loads

2.3 Material Strength

Concrete Compressive Strength (f'_c)

Columns, Beams, Slabs, Shear Walls	24.24 MPa (3,515 psi)
Foundation	22.61 MPa (3,279 psi)

Steel Reinforcement Bars for Concrete will be Deformed Bars of Billet Steel

Yield Strength (f_y)	425 MPa (61,641 psi)
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2.4 Load Combinations

For ultimate strength design, the structure and all lateral resisting elements shall be designed to resist the most critical effects of load combinations defined in the NSCP and generated in the ETABS 20 software. Moreover, wind loads and earthquake loads are considered in this particular design calculation report.

The following are the load combinations used for Ultimate Strength Design:

- $1.4(D+F)$
- $1.2(D+F+T) + 1.6(L+H) + 0.5(L_r \text{ or } R)$
- $1.2D + 1.6(L_r \text{ or } R) + (f_1L \text{ or } 0.8W)$
- $1.2D + 1.6W + f_1L + 0.5(L_r \text{ or } R)$
- $1.2D + 1.0E + f_1L$
- $0.9D + 1.6W + 1.6H$
- $0.9D + 1.0E + 1.6h$

Where:

D = Dead Load

L = Live Load

E = Earthquake Load (One way and Orthogonal Direction)

L_r = Roof Live Load

F = Loads due to weight and pressure of fluids

W = Wind Load

3 Design Standards and References

3.1 Codes and Practices

The following references have been used for the structural design:

- National Structural Code of the Philippines (NSCP) 2015
- National Building Code of the Philippines (NBCP)

3.2 Computer Programs

The following computer programs were used for the analysis and design of the structure:

- Calculation Spreadsheets developed from Microsoft Office Programs – General calculation tools to design and evaluate structural elements.
- AutoCAD by Autodesk – Final foundation design drawing
- ETABS 20 – Numerical Modelling and Simulation for the extraction of governing load factored and unfactored forces used to design the foundations

4 Modelling and Analysis

4.1 Structural Model

The mathematical model of the Annex Building (Maysan National Highschool) is modelled through the ETABS 20 software, whereas all the aforementioned loads and load cases are applied to its appropriate location. Shown in Figure 4.1.1 is the plan view of the mathematical model where the unique names of the columns can be observed. Meanwhile, Figure 4.1.2 shows the 3-D model of the Annex Building.



Figure 4.1.1. Plan View

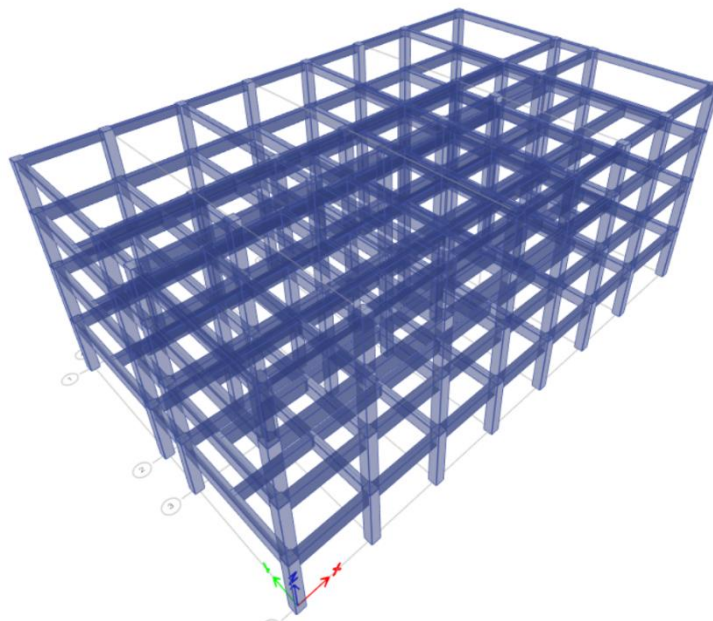


Figure 4.1.2. 3D Model

4.2 Structural Analysis

The joint reactions of the ground floor columns were the forces which will be used for the design of the different types of shallow foundation.

4.2.1 Exterior Columns

As shown in Table 4.2.1.1., the maximum factored axial force for the exterior columns is 966.493 kN, which was generated from the column with ID 140. The generated force will also be used for the design of the typical square footings identified in Figure 4.2.1.

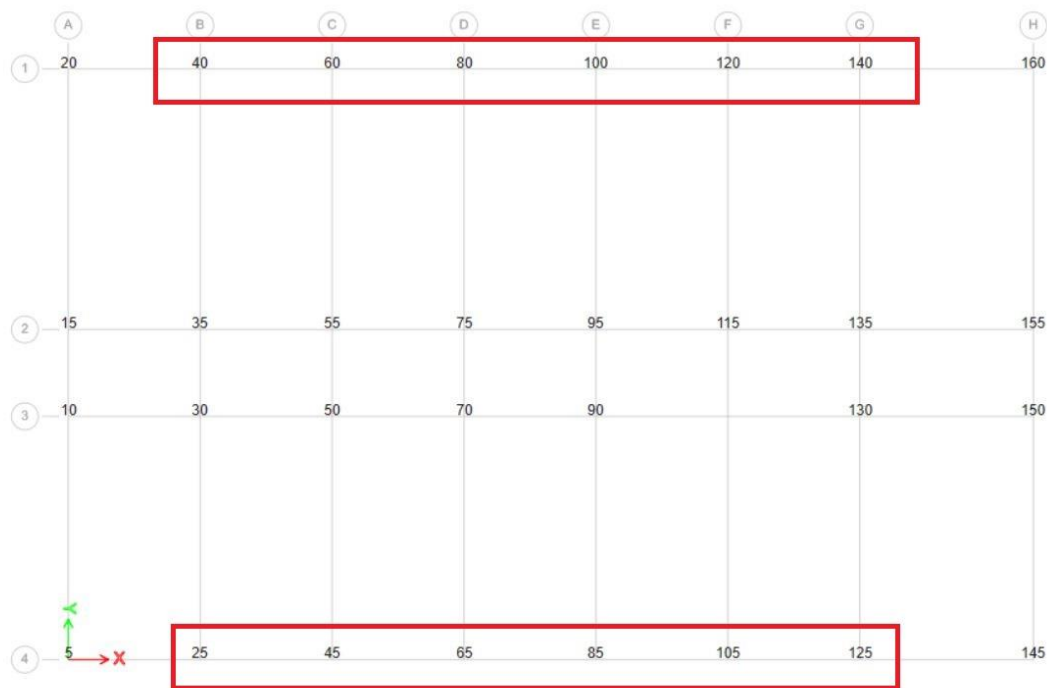


Figure 4.2.1.1. Exterior columns of the structure

Table 4.2.1.1. Factored and unfactored loads

	Type	ID	Output	Force (kN)
Factored	Fx	40	ULS04	120.294
	Fy	125	ULS06	107.907
	Fz	140	ULS02	966.493
Unfactored	Fx	40	SLS04	85.8479
	Fy	125	SLS06	81.9441
	Fz	140	SLS02	755.047

4.2.2 Corner Columns

As shown in Table 4.2.2.1, the maximum factored axial force for the corner columns is 772.899 kN, which was generated from the column with ID 160. The generated force will also be used for the design of the typical rectangular footings identified in Figure 4.2.2.1.



Figure 4.2.2.1. Corner columns of the structure

Table 4.2.2.1. Factored and unfactored loads

	Type	ID	Output	Force
Factored	Fx	20	ULS04	93.4883
	Fy	145	ULS06	107.92
	Fz	160	ULS07	772.899
Unfactored	Fx	20	SLS04	67.2848
	Fy	145	SLS06	80.3885
	Fz	160	SLS07	670.727

4.2.3 Interior Columns

As shown in Table. 4.2.3.1, the max factored axial force for the interior columns is 1295.41 kN which was generated from the column with ID 115. The generated force will also be used for the design of the isolated square footing of the structure in Figure 4.2.3.1.

A. Isolated Column



Figure 4.2.3.1. Isolated square footing

Table 4.2.3.1. Factored and unfactored loads

	Type	ID	Output	Force
Factored	Fx	115	ULS04	114.24
	Fy	115	ULS06	123.699
	Fz	115	ULS02	1295.41
Unfactored	Fx	115	SLS04	81.5281
	Fy	115	SLS06	89.3227
	Fz	115	SLS02	992.395

B. Column with an Adjacent Column

As shown in Table 4.2.3.2, the max factored axial force for the columns with an adjacent column is 1838.16 kN which was generated from the column with ID 115. Moreover, the adjacent column identified as ID 135 generated a factored axial force of 1612.1 kN. The generated loads from both columns will then be used for the design of the combined footings of the structure in Figure 4.2.3.2.



Figure 4.2.3.2. Columns with an adjacent column

Table 4.2.3.2. Factored and unfactored loads

	Type	ID	Output	Force
Factored	Fx	90	ULS08	144.346
	Fy	155	ULS06	176.975
	Fz	130	ULS06	1838.16
Unfactored	Fx	90	SLS08	107.226
	Fy	155	SLS06	129.571
	Fz	130	SLS06	1628.96

Opposite Column				
	Type	ID	Output	Force
Factored	Fx	135	ULS04	111.309
	Fy	135	ULS06	171.99
	Fz	135	ULS09	1612.1
Unfactored	Fx	135	SLS04	80.1074
	Fy	135	SLS06	127.144
	Fz	135	SLS09	1393.25

5 Foundation Design

The proposed foundations to the structure will be composed of three different types of shallow foundations: square, rectangular, and combined footings. The structural consultants developed the design computation through a Microsoft Excel worksheet and design illustrations through the Autodesk software. Moreover, the calculations performed for all footings are based on the NSCP 2015.

5.1 Square Footing

A. Typical Footing

The footprint dimensions of the typical square footing were determined using the allowable soil pressure (q_a), the unfactored axial load (P), the unit weight of the concrete (γ_c), and the assumed total depth of the embedment of the footing (D).

- $$\begin{aligned} \bullet \text{ Weight of Footing, } W_f &= (\gamma_{concrete})(D)(B^2) \\ &= (23.6)(1.5)(B^2) \\ &= 35.4B^2 \end{aligned}$$
- $$\begin{aligned} \bullet \text{ Base of Footing, } B &= \sqrt{\frac{P}{q_a - (\gamma_{concrete})(D)}} \\ &= \sqrt{\frac{755.047}{400 - 35.4}} \\ &= 1.4391 \text{ m} \\ &\approx 1.5 \text{ m} \end{aligned}$$

To determine the thickness, the ultimate two-way shear capacity of the concrete was equated to the maximum applied shear with respect to the axial load and shear force the two-way shear plane.

$$\begin{aligned} V_{uc} &= \phi V_{nc} \\ \left(\frac{B^2 - (c + d)^2}{B^2} \right) \left(\sqrt{\left(\frac{P_u}{4} + \frac{M_u}{c + d} \right)^2 + \left(\frac{V_u}{2} \right)^2} \right) &= \phi \left(\frac{1}{3} \right) (d)(c + d) (\sqrt{f'_c}) \\ \left(\frac{1.5^2 - (0.45 + d)^2}{1.5} \right) \left(\sqrt{\left(\frac{966.4928}{4} + 0 \right)^2 + \left(\frac{223.93}{2} \right)^2} \right) &= \phi \left(\frac{1}{3} \right) (d)(0.45 + d) (\sqrt{22.61'}) \left(\frac{1}{1000} \right) \end{aligned}$$

- Effective Depth of Rebars, d $= 239.57$
- Footing thickness, T $= d + d_b + cc$
 $= 239.57 + 16 + 75$
 $= 330.57 \text{ mm}$
 $\approx 350 \text{ mm}$
- New Effective Depth, d $= T - cc - d_b$
 $= 350 - 75 - 16$
 $= 259 \text{ mm}$

Check design for One-way Shear if the ultimate capacity can withstand the maximum applied one-way shear with respect to the factored axial load and shear force. As shown below, the capacity, V_{nc} , is greater than the V_{uc} , thus, the effective depth is acceptable.

- One-way shear capacity, V_{nc} $= \phi \left(\frac{1}{6} \right) (b_w)(d) (\sqrt{f_c'})$
 $= 0.9 \left(\frac{1}{6} \right) (2)(1500)(259)(\sqrt{22.61}) \frac{1}{1000}$
 $= 615.77 \text{ kN}$
- Applied one-way shear, V_{uc} $= \left(\frac{B - (c + 2d)}{B} \right) \left(\sqrt{\left(\frac{P_u}{4} + \frac{M_u}{c + d} \right)^2 + \left(\frac{V_u}{2} \right)^2} \right)$
 $= \left(\frac{1.5 - 0.45 + 2(0.259)}{1.5} \right)$
 $\left(\sqrt{\left(\frac{966.49}{4} + 0 \right)^2 + \left(\frac{120.29}{2} \right)^2} \right)$
 $= 94.76 \text{ kN}$

Moving on to the design for flexure, the determined number of reinforcements with respect to the maximum applied moment is considered to be the same in both the transverse and longitudinal directions. Also, as shown, the minimum area of reinforcements governs thus, it is the amount of steel to be supplied to the square footing. Moreover, the supplied

development length is greater than the required development length, thus, the chosen bar size is adequate. The calculations are as followed:

- Cantilever Distance, l

$$= \frac{B - c}{2}$$

$$= \frac{1500 - 450}{2}$$

$$= 525 \text{ mm}$$
- Factored Bending Moment, M_{uc}

$$= \frac{Pl^2}{2B}$$

$$= \frac{(966.49)(525)^2}{2(1500)}$$

$$= 88.7965 \text{ kN} - m$$
- Required Amount if Steel, A_{sreq}

$$= \left(\frac{f'c b}{1.176f_y} \right) \left(d - \sqrt{d^2 - \frac{2.353M_u}{\phi f'c b}} \right)$$

$$= \left(\frac{22.61 (1500)}{1.176(425)} \right)$$

$$\left(259 - \sqrt{259^2 - \frac{2.353(88.797 \times 10^6)}{0.9(22.61)(1500)}} \right)$$

$$= 920.83 \text{ mm}^2$$
- Min Required Amount if Steel, A_{smin}

$$= \frac{(0.0018)(420)(A_g)}{f_y}$$

$$= \left(\frac{(0.0018)(420)(1500)(350)}{425} \right)$$

$$= 933.88 \text{ mm}^2, \text{ governs}$$

- Number of Reinforcing bars, n

$$= \frac{4As}{\pi d_b^2}$$

$$= \frac{4(933.88)}{\pi(16^2)}$$

$$= 5$$
- Required O.C. Spacing, $S_{o.c.}$

$$= \frac{B - 2cc - d_b}{n - 1}$$

$$= \frac{1500 - 2(75) - 16}{5 - 1}$$

$$= 333.5 \text{ mm}$$

$$\approx 325 \text{ mm}$$
- Required Clear Spacing, $S_{c.s.}$

$$= s_{o.c.} - d_b$$

$$= 325 - 16$$

$$= 309 \text{ mm}$$
- Concrete cover distance, cc

$$= \frac{B - nd_b - s_{c.s.}(n - 1)}{2}$$

$$= \frac{1500 - 5(16) - 309(5 - 1)}{2}$$

$$= 92 \text{ mm}$$
- Supplied Development Length, l_s

$$= l - cc$$

$$= 525 - 92$$

$$= 433 \text{ mm}$$
- C_b

$$= \text{smaller} \left\{ \begin{array}{l} cc + \frac{d_b}{2} \\ \frac{S_{o.c.}}{2} \end{array} \right.$$

$$= \text{smaller} \left\{ \begin{array}{l} 92 + \frac{16}{2} \\ \frac{325}{2} \end{array} \right.$$

$$= 83 \text{ mm}$$

- $$\bullet \quad \frac{c_b + K_{tr}}{d_b} = \frac{83 + 0}{16}$$

$$= 5.19 \leq 2.5$$

$$= \therefore \text{use } 2.5$$
- $$\bullet \quad \varphi_s = \begin{cases} 25\text{mm } \emptyset \text{ or larger,} & 1.0 \\ 20\text{mm } \emptyset \text{ or smaller,} & 0.8 \end{cases}$$

$$= 0.8$$
- $$\bullet \quad \text{Required Development Length, } l_d = \left(\frac{1}{1.1} \right) \left(\frac{f_y}{(\lambda) \sqrt{f'_c}} \right) \left(\frac{(\varphi_t)(\varphi_e)(\varphi_s)}{\frac{c_b + K_{tr}}{d_b}} \right) (d_b)$$

$$= \left(\frac{1}{1.1} \right) \left(\frac{425}{(1) \sqrt{22.61}} \right) \left(\frac{(1)(1)(0.8)}{2.5} \right) (16)$$

$$= 416.02 \text{ mm}$$

B. Isolated Interior Footing

The footprint dimensions of the isolated interior square footing were determined using the allowable soil pressure (q_a), the unfactored axial load (P), the unit weight of the concrete (γ_c), and the assumed total depth of the embedment of the footing (D).

- $$\bullet \quad \text{Weight of Footing, } W_f = (\gamma_{concrete})(D)(B^2)$$

$$= (23.6)(1.5)(B^2)$$

$$= 35.4B^2$$
- $$\bullet \quad \text{Base of Footing, } B = \sqrt{\frac{P}{q_a - (\gamma_{concrete})(D)}}$$

$$= \sqrt{\frac{992.395}{400 - 35.4}}$$

$$= 1.6498 \text{ m}$$

$$\approx 1.75 \text{ m}$$

To determine the thickness, the ultimate two-way shear capacity of the concrete was equated to the maximum applied shear with respect to the axial load and shear force the two-way shear plane.

$$V_{uc} = \phi V_{nc}$$

$$\left(\frac{B^2 - (c + d)^2}{B^2} \right) \left(\sqrt{\left(\frac{P_u}{4} + \frac{M_u}{c + d} \right)^2 + \left(\frac{V_u}{2} \right)^2} \right) = \phi \left(\frac{1}{3} \right) (d)(c + d) (\sqrt{f'_c})$$

$$\begin{aligned} & \left(\frac{1.75^2 - (0.45 + d)^2}{1.75^2} \right) \left(\sqrt{\left(\frac{1295.41}{4} + 0 \right)^2 + \left(\frac{123.70}{2} \right)^2} \right) \\ & = \phi \left(\frac{1}{3} \right) (d)(0.45 + d)(\sqrt{22.61'}) \left(\frac{1}{1000} \right) \end{aligned}$$

- Effective Depth of Rebars, d = 301.14
- Footing thickness, T = d + d_b + cc

$$= 301.14 + 16 + 75$$

$$= 392.14 \text{ mm}$$

$$\approx 400 \text{ mm}$$
- New Effective Depth, d = T - cc - d_b

$$= 400 - 75 - 16$$

$$= 309 \text{ mm}$$

Check design for One-way Shear if the ultimate capacity can withstand the maximum applied one-way shear with respect to the factored axial load and shear force. As shown below, the capacity, V_{nc} , is greater than the V_{uc} , thus, the effective depth is acceptable.

- One-way shear capacity, V_{nc} = $\phi \left(\frac{1}{6} \right) (b_w)(d) (\sqrt{f'_c})$

$$= 0.9 \left(\frac{1}{6} \right) (2)(1750)(309)(\sqrt{22.61}) \frac{1}{1000}$$

$$= 857.09 \text{ kN}$$
- Applied one-way shear, V_{uc} = $\left(\frac{B - (c + 2d)}{B} \right) \left(\sqrt{\left(\frac{P_u}{4} + \frac{M_u}{c + d} \right)^2 + \left(\frac{V_u}{2} \right)^2} \right)$

$$= \left(\frac{1.75 - 0.45 + 2(0.309)}{1.75} \right)$$

$$\left(\sqrt{\left(\frac{1295.41}{4} + 0 \right)^2 + \left(\frac{123.70}{2} \right)^2} \right)$$

$$= 128.49 \text{ kN}$$

In terms of the design for flexure, the determined number of reinforcements with respect to the maximum applied moment is considered to be the same in both the transverse and longitudinal directions. Moreover, the supplied development length is greater than the required development length, thus, the chosen bar size is adequate. The calculations are performed as follows:

- Cantilever Distance, l

$$= \frac{B - c}{2}$$

$$= \frac{1750 - 450}{2}$$

$$= 650 \text{ mm}$$
- Factored Bending Moment, M_{uc}

$$= \frac{Pul^2}{2B}$$

$$= \frac{(1295.41)(600)^2}{2(1750)}$$

$$= 156.374 \text{ kN} - m$$
- Required Amount if Steel, A_{sreq}

$$= \left(\frac{f'c b}{1.176f_y} \right) \left(d - \sqrt{d^2 - \frac{2.353M_u}{\phi f'c b}} \right)$$

$$= \left(\frac{22.61 (1750)}{1.176(425)} \right)$$

$$\left(309 - \sqrt{309^2 - \frac{2.353(156.374 \times 10^6)}{0.9(22.61)(1750)}} \right)$$

$$= 1399.23 \text{ mm}^2, \text{ governs}$$
- Min Required Amount if Steel, A_{smin}

$$= \frac{(0.0018)(420)(A_g)}{f_y}$$

$$= \left(\frac{(0.0018)(420)(1750)(400)}{425} \right)$$

$$= 1245.18 \text{ mm}^2$$

- Number of Reinforcing bars, n

$$= \frac{4As}{\pi d_b^2}$$

$$= \frac{4(1399.23)}{\pi(20^2)}$$

$$= 5$$
- Required O.C. Spacing, $S_{o.c.}$

$$= \frac{B - 2cc - d_b}{n - 1}$$

$$= \frac{1750 - 2(75) - 16}{5 - 1}$$

$$= 395 \text{ mm}$$

$$\approx 375 \text{ mm}$$
- Required Clear Spacing, $S_{c.s.}$

$$= s_{o.c.} - d_b$$

$$= 375 - 20$$

$$= 355 \text{ mm}$$
- Concrete cover distance, cc

$$= \frac{B - nd_b - s_{c.s.}(n - 1)}{2}$$

$$= \frac{1750 - 5(20) - 355(5 - 1)}{2}$$

$$= 115 \text{ mm}$$
- Supplied Development Length, l_s

$$= l - cc$$

$$= 650 - 115$$

$$= 535 \text{ mm}$$
- C_b

$$= \text{smaller} \left\{ \begin{array}{l} cc + \frac{d_b}{2} \\ \frac{S_{o.c.}}{2} \end{array} \right.$$

$$= \text{smaller} \left\{ \begin{array}{l} 115 + \frac{16}{2} \\ \frac{395}{2} \end{array} \right.$$

$$= 125 \text{ mm}$$

$$\begin{aligned}
\bullet \quad \frac{c_b + K_{tr}}{d_b} &= \frac{125 + 0}{16} \\
&= 7.81 \leq 2.5 \\
&= \therefore \text{use } 2.5 \\
\bullet \quad \varphi_s &= \begin{cases} 25\text{mm } \emptyset \text{ or larger,} & 1.0 \\ 20\text{mm } \emptyset \text{ or smaller,} & 0.8 \end{cases} \\
&= 0.8 \\
\bullet \quad \text{Required Development Length, } l_d &= \left(\frac{1}{1.1} \right) \left(\frac{f_y}{(\lambda) \sqrt{f'_c}} \right) \left(\frac{(\varphi_t)(\varphi_e)(\varphi_s)}{\frac{c_b + K_{tr}}{d_b}} \right) (d_b) \\
&= \left(\frac{1}{1.1} \right) \left(\frac{425}{(1) \sqrt{22.61}} \right) \left(\frac{(1)(1)(0.8)}{2.5} \right) (20) \\
&= 520.03 \text{ mm}
\end{aligned}$$

5.2 Rectangular Footing

The footprint dimensions of the typical rectangular footing were determined using the allowable soil pressure (q_a), the unfactored axial load (P), the unit weight of the concrete (γ_c), the assumed total depth of the embedment of the footing (D), and the assumed base of the footing (B). The area of the foundation (A) is calculated from the formulated equation of the weight the footing, which would then provide the value of the length of the foundation (L) considering an assumed base dimension.

- $$\begin{aligned} \bullet \text{ Weight of Footing, } W_f &= (\gamma_{concrete})(D)(A) \\ &= (23.6)(1.5)(A) \\ &= 35.4A \end{aligned}$$
- $$\begin{aligned} \bullet \text{ Area of Foundation, } A &= \frac{P}{q_a - (\gamma_{concrete})(D)} \\ &= \frac{670.727}{400 - (23.6)(1.5)} \\ &= 1.8396 \text{ m}^2 \end{aligned}$$
- $$\begin{aligned} \bullet \text{ Length of Foundation, } L &= \frac{A}{B} \\ &= \frac{1.8396}{1.25} \\ &= 1.4717 \text{ m} \\ &\approx 1.5 \text{ m} \end{aligned}$$

Where: B is assumed to be 1.25

To determine the thickness (T) and effective depth (d), the ultimate two-way shear capacity of the concrete was equated to the maximum applied shear with respect to the axial load and shear force the two-way shear plane. An assumed diameter of steel reinforcement (d_b) was used to calculate the thickness of the foundation.

$$V_{uc} = \phi V_{nc}$$

$$\left(\frac{B^2 - (c + d)^2}{B^2} \right) \left(\sqrt{\left(\frac{P_u}{4} + \frac{M_u}{c + d} \right)^2 + \left(\frac{V_u}{2} \right)^2} \right) = \phi \left(\frac{1}{3} \right) (d)(c + d) (\sqrt{f'_c})$$

$$\left(\frac{1.5^2 - (0.45 + d)^2}{1.5} \right) \left(\sqrt{\left(\frac{772.8994}{4} + 0 \right)^2 + \left(\frac{80.3885}{2} \right)^2} \right)$$

$$= \phi \left(\frac{1}{3} \right) (d)(0.45 + d)(\sqrt{22.61'}) \left(\frac{1}{1000} \right)$$

- $$\bullet \text{ Effective Depth of Rebars, } d = 198.56$$
- $$\begin{aligned} \bullet \text{ Footing thickness, } T &= d + d_b + cc \\ &= 198.56 + 16 + 75 \end{aligned}$$

$$= 289.56 \text{ mm}$$

$$\approx 300 \text{ mm}$$

- New Effective Depth, d

$$= T - cc - d_b$$

$$= 300 - 75 - 16$$

$$= 209 \text{ mm}$$

The design of the rectangular foundation in terms of one-way shear was performed to check and determine if the ultimate capacity can withstand the maximum applied one-way shear with respect to the factored axial load and shear force. As shown below, the capacity (V_{nc}), is greater than the demand (V_{uc}), and thus, the effective depth is acceptable.

- One-way shear capacity, V_{nc}

$$= \phi \left(\frac{1}{6} \right) (b_w)(d) (\sqrt{f_c'})$$

$$= 0.75 \left(\frac{1}{6} \right) (2)(1250)(209)(\sqrt{22.61}) \frac{1}{1000}$$

$$= 393.39 \text{ kN}$$

- Applied one-way shear, V_{uc}

$$= \left(\frac{B - (c + 2d)}{B} \right) \left(\sqrt{\left(\frac{P_u}{4} + \frac{M_u}{c + d} \right)^2 + \left(\frac{V_u}{2} \right)^2} \right)$$

$$= \left(\frac{1.5 - 0.45 + 2(0.209)}{1.5} \right)$$

$$\left(\sqrt{\left(\frac{772.8994}{4} + 0 \right)^2 + \left(\frac{80.3885}{2} \right)^2} \right)$$

$$= 63.61 \text{ kN}$$

In terms of the design for flexure, the number of reinforcements determined with respect to the maximum applied moment is different for both transverse and longitudinal directions. Thus, the design for flexure is required for both the longitudinal and transverse steel reinforcement. As shown, the calculated required area of reinforcements governs thus, it is the amount of steel to be supplied to the critical cross section of the square footing. Moreover, the supplied development length is greater than the required development length, thus, the chosen bar size is adequate. The calculations are as followed:

A. Longitudinal Steel

- Cantilever Distance, l

$$= \frac{B - c}{2}$$

$$= \frac{1500 - 450}{2}$$

$$= 525 \text{ mm}$$
- Factored Bending Moment, M_{uc}

$$= \frac{Pul^2}{2B}$$

$$= \frac{(772.8994)(525)^2}{2(1500)}$$

$$= 71.0101 \text{ kN} - m$$
- Required Amount of Steel, A_{sreq}

$$= \left(\frac{f'c b}{1.176f_y} \right) \left(d - \sqrt{d^2 - \frac{2.353M_u}{\phi f'c b}} \right)$$

$$= \left(\frac{22.61 (1250)}{1.176(425)} \right) \left(209 - \sqrt{209^2 - \frac{2.353(71.0101 \times 10^6)}{0.9(22.61)(1250)}} \right)$$

$$= 977.98 \text{ mm}^2, \text{ governs}$$
- Min Required Amount of Steel, A_{smin}

$$= \frac{(0.0018)(420)(A_g)}{f_y}$$

$$= \left(\frac{(0.0018)(420)(1250)(300)}{425} \right)$$

$$= 667.06 \text{ mm}^2$$
- Number of Reinforcing bars, n

$$= \frac{4A_s}{\pi d_b^2}$$

$$= \frac{4(977.98)}{\pi(12^2)}$$

$$= 9$$
- Required O.C. Spacing, $S_{o.c.}$

$$= \frac{B - 2c - d_b}{n - 1}$$

$$\begin{aligned}
 &= \frac{1250 - 2(75) - 12}{9 - 1} \\
 &= 136 \text{ mm} \\
 &\approx 125 \text{ mm} \\
 \bullet \text{ Required Clear Spacing, } S_{c.s.} &= s_{o.c.} - d_b \\
 &= 125 - 12 \\
 &= 113 \text{ mm} \\
 \bullet \text{ Concrete cover distance, } cc &= \frac{B - nd_b - s_{c.s.}(n - 1)}{2} \\
 &= \frac{1250 - 9(12) - 113(9 - 1)}{2} \\
 &= 119 \text{ mm} \\
 \bullet \text{ Supplied Development Length, } l_s &= l - cc \\
 &= 525 - 119 \\
 &= 406 \text{ mm} \\
 \bullet C_b &= \text{smaller} \left\{ \begin{array}{l} cc + \frac{d_b}{2} \\ \frac{s_{o.c.}}{2} \end{array} \right. \\
 &= \text{smaller} \left\{ \begin{array}{l} 117 + \frac{12}{2} \\ \frac{125}{2} \end{array} \right. \\
 &= 62.5 \text{ mm} \\
 \bullet \frac{c_b + K_{tr}}{d_b} &= \frac{62.5 + 0}{12} \\
 &= 5.21 \leq 2.5 \\
 &= \therefore \text{use } 2.5 \\
 \bullet \varphi_s &= \begin{cases} 25\text{mm } \emptyset \text{ or larger,} & 1.0 \\ 20\text{mm } \emptyset \text{ or smaller,} & 0.8 \end{cases} \\
 &= 0.8
 \end{aligned}$$

- Required Development Length, l_d

$$= \left(\frac{1}{1.1} \right) \left(\frac{f_y}{(\lambda) \sqrt{f'_c}} \right) \left(\frac{(\phi_t)(\phi_e)(\phi_s)}{\frac{c_b + K_{tr}}{d_b}} \right)$$

$$= \left(\frac{1}{1.1} \right) \left(\frac{425}{(1) \sqrt{22.61}} \right) \left(\frac{(1)(1)(0.8)}{\frac{2.5}{12}} \right)$$

$$= 312.02 \text{ mm}$$

B. Transverse Steel

- Cantilever Distance, l

$$= \frac{B - c}{2}$$

$$= \frac{1250 - 450}{2}$$

$$= 400 \text{ mm}$$
- Factored Bending Moment, M_{uc}

$$= \frac{Pul^2}{2B}$$

$$= \frac{(772.8994)(400)^2}{2(1250)}$$

$$= 49.466 \text{ kN} - \text{m}$$

- Required Amount of Steel, $A_{s_{req}}$

$$= \left(\frac{f'c b}{1.176f_y} \right) \left(d - \sqrt{d^2 - \frac{2.353M_u}{\phi f'c b}} \right)$$

$$= \left(\frac{22.61 (1500)}{1.176(425)} \right)$$

$$\left(209 - \sqrt{209^2 - \frac{2.353(49.466 \times 10^6)}{0.9(22.61)(1500)}} \right)$$

$$= 668.157 \text{ mm}^2$$
- Min Required Amount of Steel, $A_{s_{min}}$

$$= \frac{(0.0018)(420)(A_g)}{f_y}$$

$$= \left(\frac{(0.0018)(420)(1500)(300)}{425} \right)$$

$$= 800.471 \text{ mm}^2, \text{ governs}$$
- Number of Reinforcing bars, n

$$= \frac{4A_s}{\pi d_b^2}$$

$$= \frac{4(800.471)}{\pi(10^2)}$$

$$= 11 \text{ bars}$$
- Required O.C. Spacing, $S_{o.c.}$

$$= \frac{B - 2cc - d_b}{n - 1}$$

$$= \frac{1500 - 2(75) - 10}{11 - 1}$$

$$= 134 \text{ mm}$$

$$\approx 125 \text{ mm}$$
- Required Clear Spacing, $S_{c.s.}$

$$= s_{o.c.} - d_b$$

$$= 125 - 10$$

$$= 115 \text{ mm}$$
- Concrete cover distance, cc

$$= \frac{B - nd_b - s_{c.s.}(n - 1)}{2}$$

$$= \frac{1500 - 11(10) - 115(11 - 1)}{2}$$

$$\begin{aligned}
 &= 120 \text{ mm} \\
 \bullet \text{ Supplied Development Length, } l_s &= l - cc \\
 &= 400 - 120 \\
 &= 280 \text{ mm} \\
 \bullet C_b &= \text{smaller} \left\{ \begin{array}{l} cc + \frac{d_b}{2} \\ \frac{S_{o.c.}}{2} \end{array} \right. \\
 &= \text{smaller} \left\{ \begin{array}{l} 117 + \frac{10}{2} \\ \frac{125}{2} \end{array} \right. \\
 &= 62.5 \text{ mm} \\
 \bullet \frac{c_b + K_{tr}}{d_b} &= \frac{62.5 + 0}{10} \\
 &= 6.25 \leq 2.5 \\
 &= \therefore \text{use } 2.5 \\
 \bullet \varphi_s &= \begin{cases} 25\text{mm } \emptyset \text{ or larger,} & 1.0 \\ 20\text{mm } \emptyset \text{ or smaller,} & 0.8 \end{cases} \\
 &= 0.8 \\
 \bullet \text{ Required Development Length, } l_d &= \left(\frac{1}{1.1} \right) \left(\frac{f_y}{(\lambda) \sqrt{f'_c}} \right) \left(\frac{(\varphi_t)(\varphi_e)(\varphi_s)}{\frac{c_b + K_{tr}}{d_b}} \right) \\
 &= \left(\frac{1}{1.1} \right) \left(\frac{425}{(1) \sqrt{22.61}} \right) \left(\frac{(1)(1)(0.8)}{\frac{2.5}{10}} \right) \\
 &= 260.01 \text{ mm}
 \end{aligned}$$

The determination and checking of the bearing strength of concrete with respect to the critical axial force of the column is performed to identify whether the designed footing will be able to carry the column or not. Therefore, to accept the design, the capacity of the footing (ϕP_n) must be greater than the demand, which is based on the critical axial load (P_u) of the column in contact with the foundation. Based on the calculations performed, the design is accepted since the condition $\phi P_n > P_u$ is met.

- $$\begin{aligned} \bullet \text{ Bearing Strength of Foundation, } \phi P_n &= \phi 0.85 f c' A_n \\ &= (0.65)(0.85)(22.61)(450^2) \left(\frac{[450+4(209)]^2}{450^2} \right) \left(\frac{1}{1000} \right) \\ &= 6994.28 \text{ kN} > (P_u = 772.8994 \text{ kN}) \end{aligned}$$

The banding width of the foundation is based on the width of the foundation (B). With the determined band width, the number of bars placed inside the band is calculated using the computed γ_s . The calculation of γ_s considers the ratio of the length and base. Furthermore, the spacing of the bars inside the band is calculated with respect to the band width. In terms of the spacing of the remaining bars placed outside the band, the calculation is performed considering the total length (L) of the foundation.

- $$\begin{aligned} \bullet \gamma_s &= \frac{2}{\beta + 1} \\ &= \frac{2}{1.2 + 1} \\ &= 0.9091 \end{aligned}$$
- $$\begin{aligned} \bullet n_{\text{band}} &= \gamma_s n \\ &= 0.9091(11) \\ &= 10 \text{ bars} \end{aligned}$$
- $$\begin{aligned} \bullet S_{\text{o.c. (inside)}} &= \frac{B - d_b}{n - 1} \\ &= \frac{1250 - 10}{10 - 1} \\ &= 137.78 \text{ mm} \end{aligned}$$
- $$\begin{aligned} \bullet n_{\text{outside}} &= n - n_{\text{band}} \\ &= 11 - 10 \\ &= 1 + 1 \text{ (to consider both sides)} \\ &= 2 \end{aligned}$$
- $$\bullet S_{\text{o.c. (outside)}} = \frac{\frac{L - B}{2} - cc}{\frac{n}{2}}$$

$$= \frac{\frac{1500 - 1250}{2} - 120}{\frac{2}{2}}$$

$$= 5 \text{ mm}$$

4.3 Combined Footing

The footprint dimensions of the typical combined footing were determined using the allowable soil pressure (q_a), the unfactored axial loads (P), the unit weight of the concrete (γ_c), and the assumed total depth of the embedment of the footing (D). The center-to-center distance of the adjacent columns were 2.5 m and is according to the plan layout. In this case, it was assumed that the additional length, addtl., l , of footing was 1.75 m and was measured from the center of the column.

- Footing Length, L

$$= \frac{(addtl.l)(P_a) + (addtl.l + c.l)(P_b)}{P_a + P_b}$$

$$= \frac{(1.75)(1628.96) + (1.75 + 0.75)(1393.25)}{1628.96 + 1393.25}$$

$$= 5.81 \text{ m}$$

$$\approx 6 \text{ m}$$
- Area of footing, A

$$= \frac{P}{q_a - (\gamma_{concrete})(D)}$$

$$= \frac{3022.2046}{400 - 35.4}$$

$$= 8.2891 \text{ m}^2$$
- Footing Base, B

$$= \frac{A}{L}$$

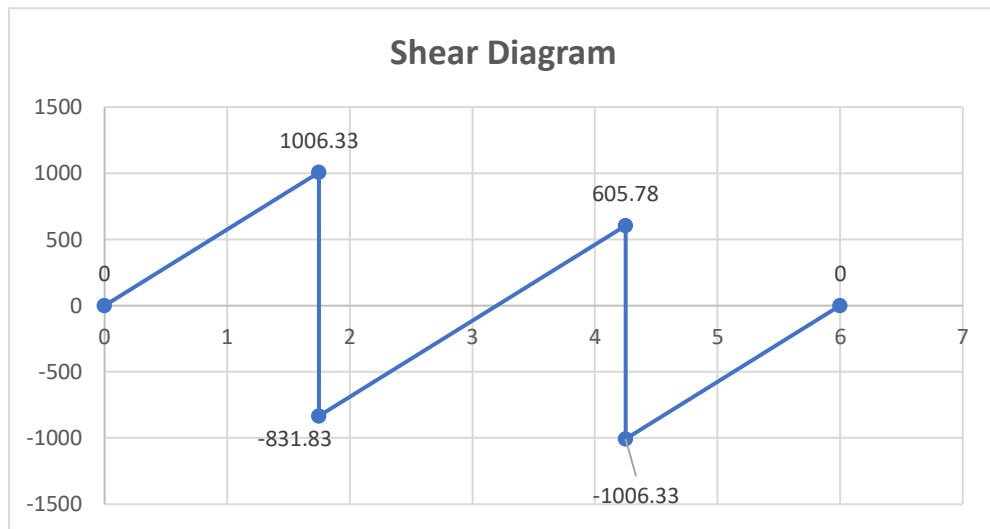
$$= \frac{8.2891}{6}$$

$$= 1.3815 \text{ m}$$

$$\approx 1.5 \text{ m}$$

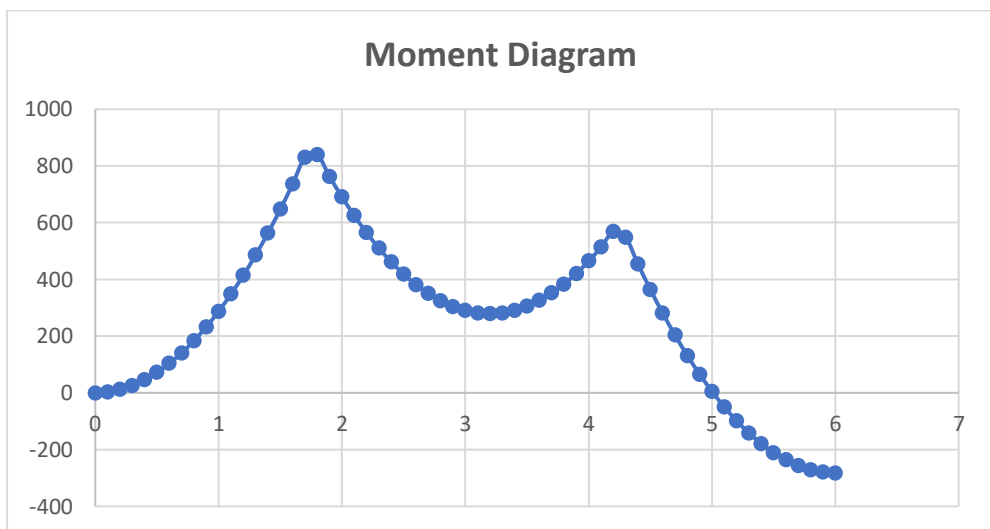
As shown below, the shear and moment diagrams were assessed in the Microsoft Excel software to check what is the value of the maximum shear and moment, and to assess where it occurred along the footing.

Shear Diagram of Footing:



- Max shear, $V_{\max}$ = 1006.326

Bending Moment Diagram of Footing



- Max Bending Moment, $M_{\max}$ = 839.66 kN – m

Moreover, the effective depth and thickness of the footing is determined from both the required dimension in terms of the applied maximum moment and from the one-way shear. Moreover, as shown below, since both thicknesses gave a net soil pressure lower than the capacity of the soil both can be used as thickness. Furthermore, it is observed that the thickness will be determined from the one-way shear because it will require a higher thickness and will already cover the required for the applied maximum moment on the footing.

- Balance Steel Ratio, ρ_b

$$= (0.85\beta_1)\left(\frac{f'_c}{f_y}\right)\left(\frac{87000}{87000 + f_y}\right)$$

$$= (0.85)(0.85)\left(\frac{22.61}{425}\right)\left(\frac{87000}{87000 + 425}\right)$$

$$= 0.0383$$
- Maximum Steel Ratio, ρ_{max}

$$= (0.626)(\rho_b)$$

$$= (0.626)(0.0383)$$

$$= 0.0239$$
- Steel Ratio, ρ

$$= (0.5)(\rho_{max})$$

$$= (0.5)(0.0239)$$

$$= 0.0119$$
- Required depth for moment, d

$$= \sqrt{\frac{M_{max}}{\phi \rho f_y \left(1 - \frac{\rho f_y}{1.7 f'_c}\right) (b)}}$$

$$= \sqrt{\frac{839.66 \times 10^6}{(0.9)(0.0119)(425)\left(1 - \frac{(0.0119)(425)}{1.7(22.61)}\right)(1500)}}$$

$$= 375.35 \text{ mm}$$

- Required depth for one-way, d

$$= \frac{V_{max} - \frac{c}{2}(w)}{V_{max} + \phi\left(\frac{1}{6}\right)(b_w)\left(\sqrt{f_c'}\right)}$$

$$= \frac{1006.33 \times 10^3 - \frac{450}{2}(575.04)}{1006.33 + 0.75\left(\frac{1}{6}\right)(1500)\left(\sqrt{22.61}\right)}$$

$$= 597.94 \text{ mm}$$
- Footing thickness, T_{moment}

$$= d + d_b + cc$$

$$= 375.35 + 20 + 75$$

$$= 460.35 \text{ mm}$$

$$\approx 475 \text{ mm}$$
- Footing thickness, T_{shear}

$$= d + d_b + cc$$

$$= 597.94 + 20 + 75$$

$$= 682.94 \text{ mm}$$

$$\approx 700 \text{ mm, governs}$$
- Net Soil Pressure_{moment}

$$= \frac{Pu_a + Pu_b}{A} + \gamma_{\text{concrete}}(0.475)$$

$$= \frac{3450.26}{(6)(1.5)} + 23.6(0.475)$$

$$= 394.23 \text{ kPa}$$
- Net Soil Pressure_{shear}

$$= \frac{Pu_a + Pu_b}{A} + \gamma_{\text{concrete}}(0.475)$$

$$= \frac{3450.26}{(6)(1.5)} + 23.6(0.7)$$

$$= 399.48 \text{ kPa}$$
- New Effective Depth, d

$$= T - cc - d_b$$

$$= 700 - 75 - 20$$

$$= 615 \text{ mm}$$

Moving on to the design for flexure, the number of main reinforcements is determined with respect to the maximum applied moment. Also, as shown, the transverse reinforcements are determined using applied load in its respective bandwidth. Moreover, the supplied

development lengths is greater than the required development lengths, thus, the chosen bar size is adequate. The calculations are as followed:

Main Reinforcement:

- Required Amount of Steel, $A_{s_{req}}$

$$= \left(\frac{f'c b}{1.176 f_y} \right) \left(d - \sqrt{d^2 - \frac{2.353 M_u}{\phi f'c b}} \right)$$

$$= \left(\frac{22.61 (1500)}{1.176 (425)} \right)$$

$$\left(615 - \sqrt{615^2 - \frac{2.353 (839.663 \times 10^6)}{0.9 (22.61) (1500)}} \right)$$

$$= 3738.39 \text{ mm}^2, \text{ governs}$$
- Min Required Amount of Steel, $A_{s_{min}}$

$$= \frac{(0.0018)(420)(A_g)}{f_y}$$

$$= \left(\frac{(0.0018)(420)(1500)(700)}{425} \right)$$

$$= 1867.76 \text{ mm}^2$$
- Number of Reinforcing bars, n

$$= \frac{4(3738.39)}{\pi 20^2}$$

$$= \frac{4(3738.39)}{\pi (20^2)}$$

$$= 12$$

- Required O.C. Spacing, $S_{o.c.}$

$$= \frac{B - 2cc - d_b}{n - 1}$$

$$= \frac{1500 - 2(75) - 20}{12 - 1}$$

$$= 120.91 \text{ mm}$$

$$\approx 100 \text{ mm}$$
- Required Clear Spacing, $S_{c.s.}$

$$= s_{o.c.} - d_b$$

$$= 100 - 20$$

$$= 80 \text{ mm}$$
- Concrete cover distance, cc

$$= \frac{B - nd_b - s_{c.s.}(n - 1)}{2}$$

$$= \frac{1500 - 12(20) - 80(12 - 1)}{2}$$

$$= 190 \text{ mm}$$
- Supplied Development Length, l_s

$$= l_{crit} - cc$$

$$= 2800 - 75$$

$$= 2725 \text{ mm}$$
- C_b

$$= \text{smaller} \left\{ \begin{array}{l} cc + \frac{d_b}{2} \\ \frac{S_{o.c.}}{2} \end{array} \right.$$

$$= \text{smaller} \left\{ \begin{array}{l} 190 + \frac{20}{2} \\ \frac{100}{2} \end{array} \right.$$

$$= 100 \text{ mm}$$
- $\frac{c_b + K_{tr}}{d_b}$

$$= \frac{100 + 0}{20}$$

$$= 5 \leq 2.5$$

$$= \therefore \text{use } 2.5$$
- ϕ_s

$$= \begin{cases} 25\text{mm } \emptyset \text{ or larger,} & 1.0 \\ 20\text{mm } \emptyset \text{ or smaller,} & 0.8 \end{cases}$$

- $\varphi_t = 1.3$
 $= 0.8$
- Required Development Length, $l_d = \left(\frac{1}{1.1}\right)\left(\frac{f_y}{(\lambda)\sqrt{f'_c}}\right)\left(\frac{(\varphi_t)(\varphi_e)(\varphi_s)}{\frac{c_b + K_{tr}}{d_b}}\right)(d_b)$
 $= \left(\frac{1}{1.1}\right)\left(\frac{425}{(1)\sqrt{22.61}}\right)\left(\frac{(1.3)(1)(0.8)}{2.5}\right)(20)$
 $= 676.04 \text{ mm}$
- Critical Moment at Columns, $Mu_{crit} = \frac{wl^2}{2}$
 $= \frac{575.04(1.525)}{2}$
 $= 668.67 \text{ kN} - m$
- Section Modulus, $s_m = \frac{I}{c}$
 $= \frac{1500(615^2)}{\frac{615}{2}}$
 $= 1134675000 \text{ mm}^3$
- Moment Capacity, $\phi Mn = \frac{\phi(0.42)(s_m)(\sqrt{f'_c})}{1000000}$
 $= \frac{0.6(0.42)(1134675000)(\sqrt{22.61})}{1000000}$
 $= 1359.64 \text{ kN} - m$, larger than the Mu_{crit} , no need to design

Transverse Reinforcement:

- Bandwidth, W

$$= addtl. l_c + \frac{c}{2} + d$$

$$= 1750 + \frac{0.45}{2} + 615$$

$$= 2590 \text{ mm}$$
- Net Factored Force, $P_{u_{crit}}$

$$= \frac{P_u}{B}$$

$$= \frac{1838.16 \times 1000}{1500}$$

$$= 1225.44 \text{ N/mm}$$
- Net Factored Moment, $M_{u_{crit}}$

$$= \frac{wl^2}{2}$$

$$= \frac{575.04}{2} \left(\frac{1500}{2} - \frac{450}{2} \right)^2$$

$$= 161.633 \text{ kN-m}$$
- Required Amount of Steel, $A_{s_{req}}$

$$= \left(\frac{f'c b}{1.176 f_y} \right) \left(d - \sqrt{d^2 - \frac{2.353 M_u}{\phi f'c b}} \right)$$

$$= \left(\frac{22.61 (2590)}{1.176 (425)} \right) \left(615 - \sqrt{615^2 - \frac{2.353 (161.633 \times 10^6)}{0.9 (22.61) (2590)}} \right)$$

$$= 3738.39 \text{ mm}^2, \text{ governs}$$
- Min Required Amount of Steel, $A_{s_{min}}$

$$= \frac{(0.0018)(420)(A_g)}{f_y}$$

$$= \left(\frac{(0.0018)(420)(2590)(700)}{425} \right)$$

$$= 3225 \text{ mm}^2$$
- Number of Reinforcing bars, n

$$= \frac{4(A_s)}{\pi d_b^2}$$

	$= \frac{4(3225)}{\pi(16^2)}$
	$= 11$
• Required O.C. Spacing, $S_{o.c.}$	$= \frac{B - 2cc - d_b}{n - 1}$
	$= \frac{2590 - 2(75) - 16}{11 - 1}$
	$= 151.5 \text{ mm}$
	$\approx 150 \text{ mm}$
• Required Clear Spacing, $S_{c.s.}$	$= s_{o.c.} - d_b$
	$= 150 - 16$
	$= 134 \text{ mm}$
• Concrete cover distance, cc	$= \frac{B - nd_b - s_{c.s.}(n - 1)}{2}$
	$= \frac{2590 - 11(16) - 134(11 - 1)}{2}$
	$= 87 \text{ mm}$
• Supplied Development Length, l_s	$= l_{crit} - cc$
	$= 2800 - 75$
	$= 2725 \text{ mm}$
• C_b	$= \text{smaller} \left\{ \begin{array}{l} cc + \frac{d_b}{2} \\ \frac{S_{o.c.}}{2} \end{array} \right.$
	$= \text{smaller} \left\{ \begin{array}{l} 87 + \frac{20}{2} \\ \frac{150}{2} \end{array} \right.$
	$= 75 \text{ mm}$
• $\frac{c_b + K_{tr}}{d_b}$	$= \frac{75 + 0}{20}$
	$= 3.75 \leq 2.5$

- φ_s

$$= \therefore \text{use } 2.5$$

$$= \begin{cases} 25\text{mm } \emptyset \text{ or larger,} & 1.0 \\ 20\text{mm } \emptyset \text{ or smaller,} & 0.8 \end{cases}$$

$$= 0.8$$
- Required Development Length, l_d

$$= \left(\frac{1}{1.1} \right) \left(\frac{f_y}{(\lambda) \sqrt{f'_c}} \right) \left(\frac{(\varphi_t)(\varphi_e)(\varphi_s)}{\frac{c_b + K_{tr}}{d_b}} \right) (d_b)$$

$$= \left(\frac{1}{1.1} \right) \left(\frac{425}{(1) \sqrt{22.61}} \right) \left(\frac{(1)(1)(0.8)}{2.5} \right) (16)$$

$$= 416.02 \text{ mm}$$
- Supplied Development Length, l_s

$$= B - \frac{c}{2} - cc$$

$$= 1500 - \frac{450}{2} - 87$$

$$= 438 \text{ mm}$$

6 Final Designs

6.1 Typical Square Footing

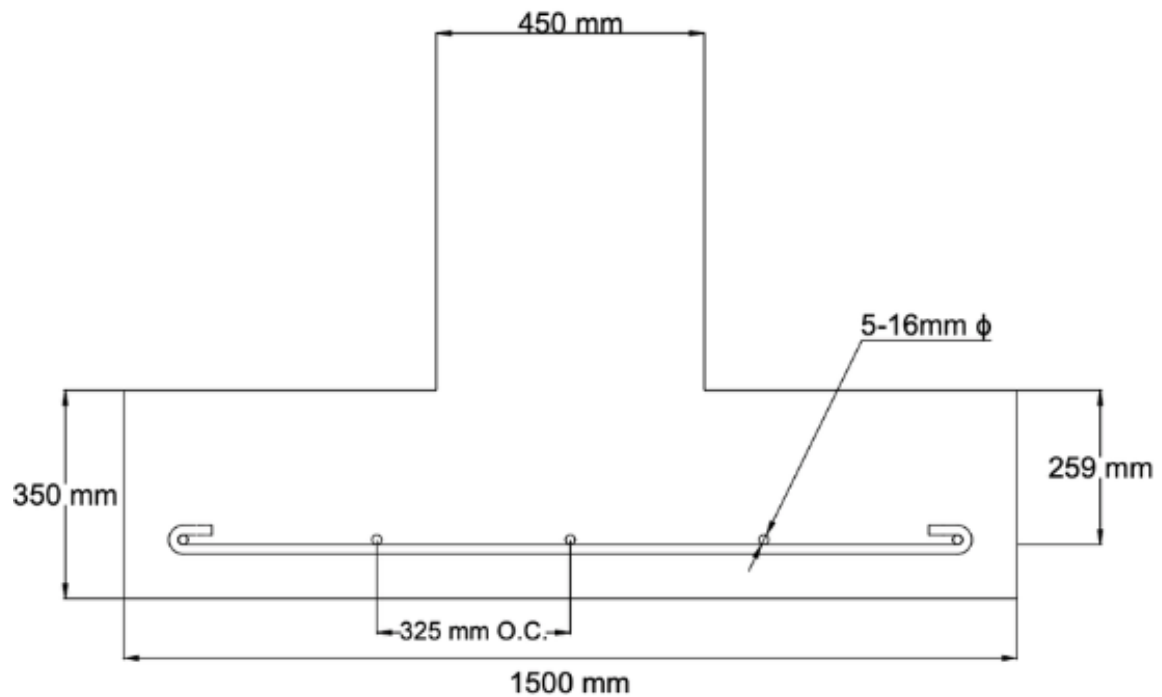


Figure 6.1.1. Cross-section View of Final Design of Typical Square Footing

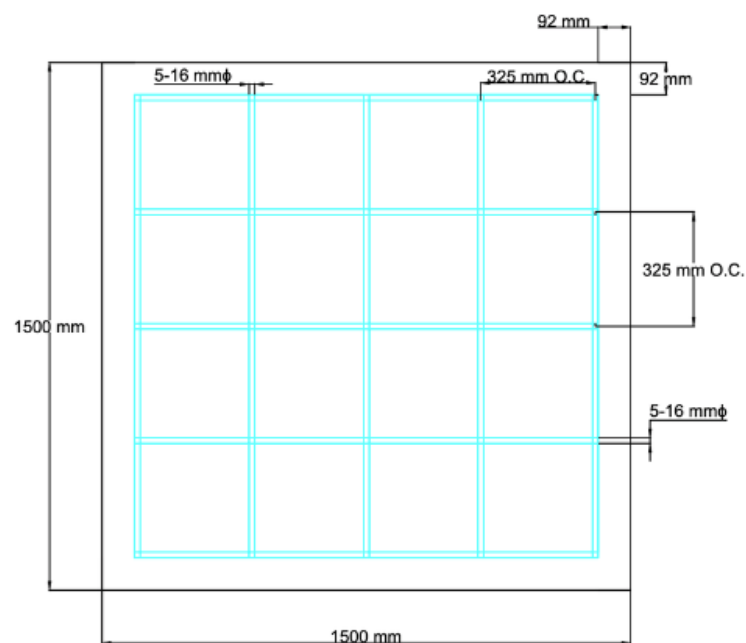


Figure 6.1.1. Top View of Final Design of Typical Square Footing

6.2 Isolated Square Footing

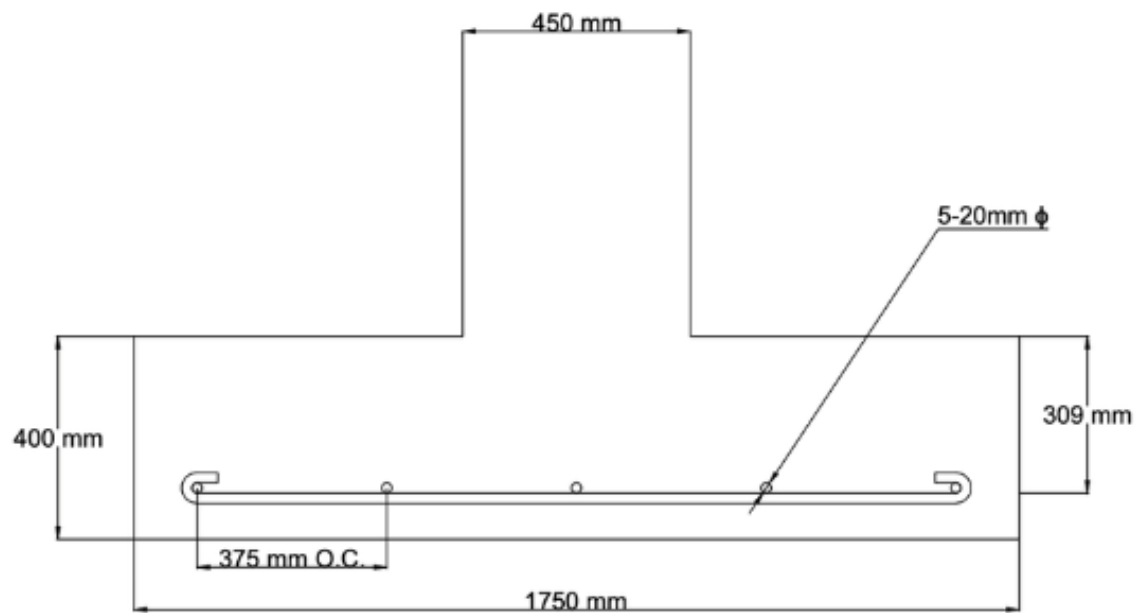


Figure 6.2.1. Cross-section View of Final Design of Isolated Square Footing

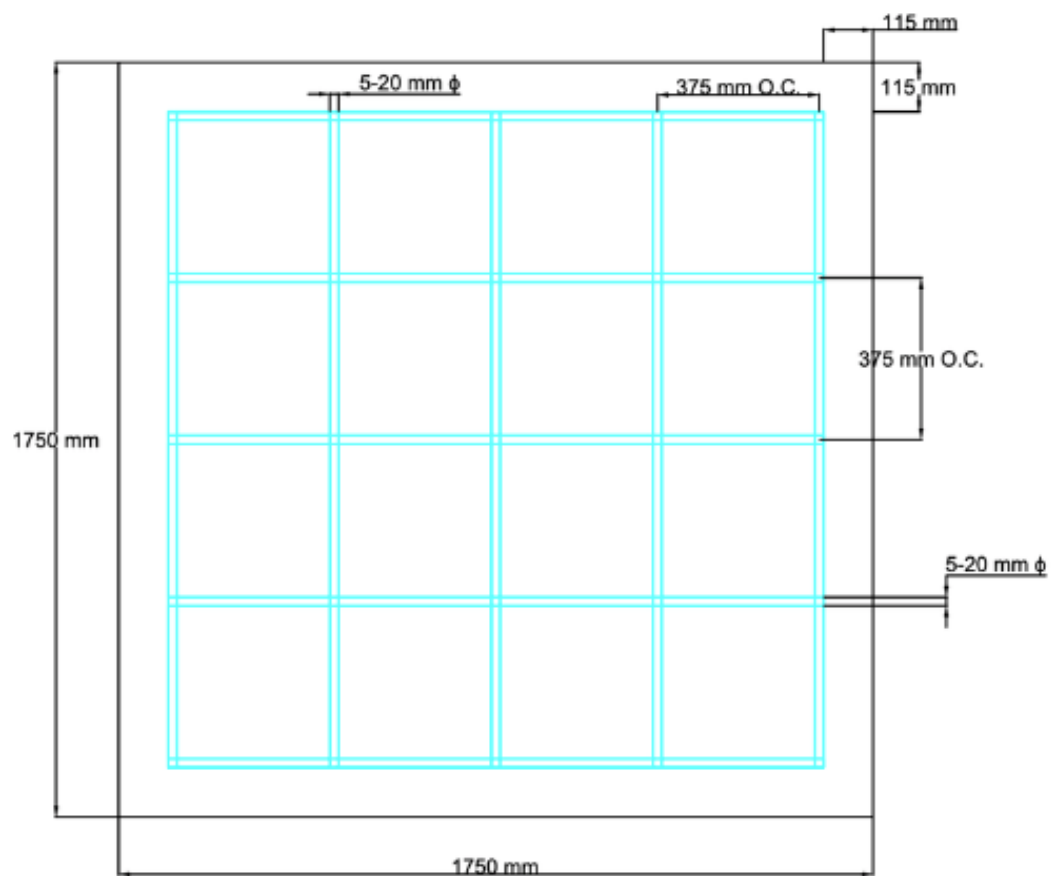


Figure 6.2.2. Top View of Final Design of Isolated Square Footing

6.3 Rectangular Footing

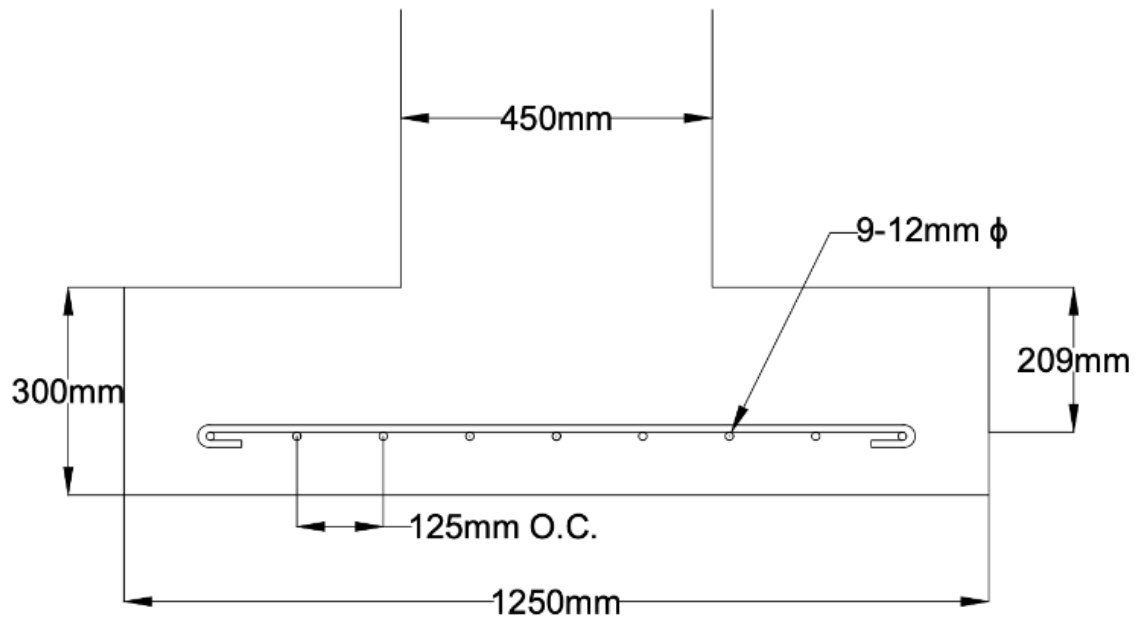


Figure 6.3.2. Transverse Cross-section View of Final Design of Rectangular Footing

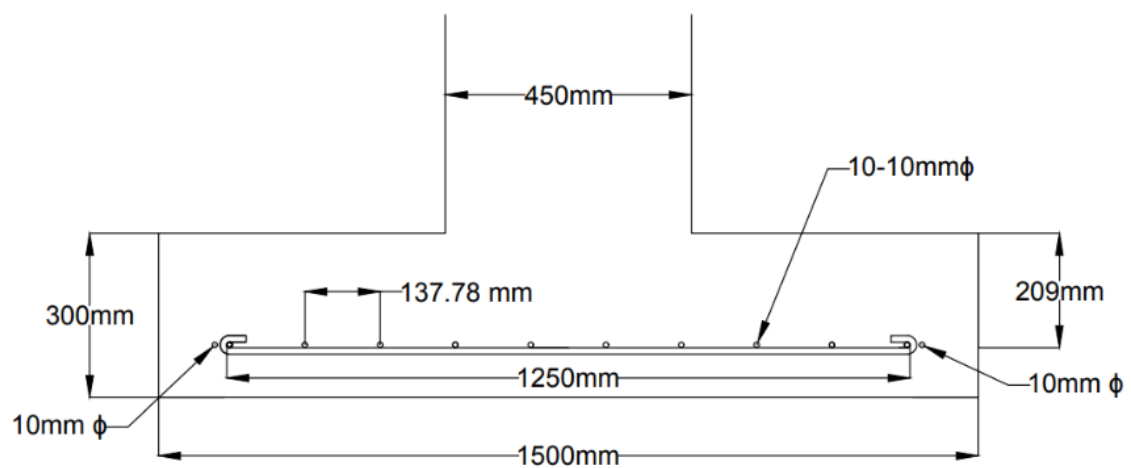


Figure 6.3.3. Longitudinal Cross-section View of Final Design of Rectangular Footing

6.4 Combined Footing

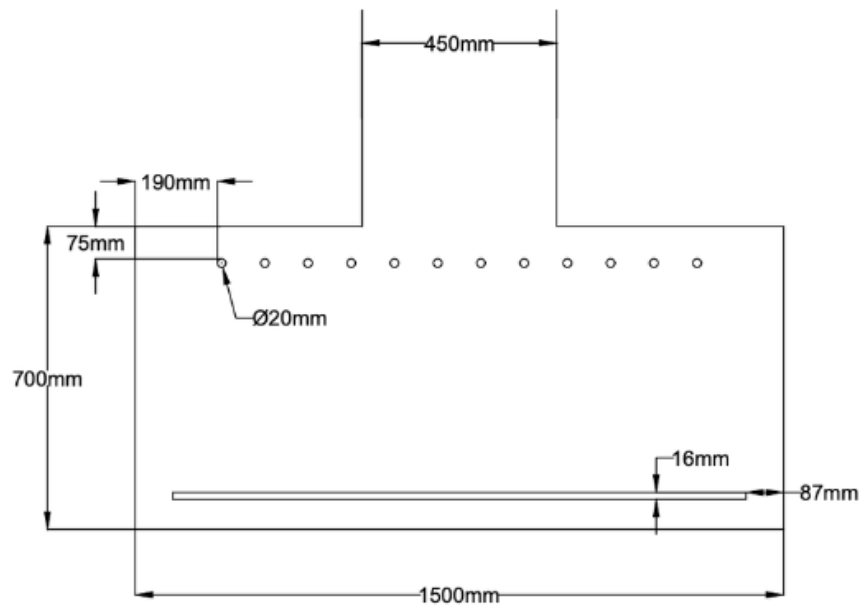


Figure 6.4.1. Transverse Cross-section View of Final Design of Combined Footing

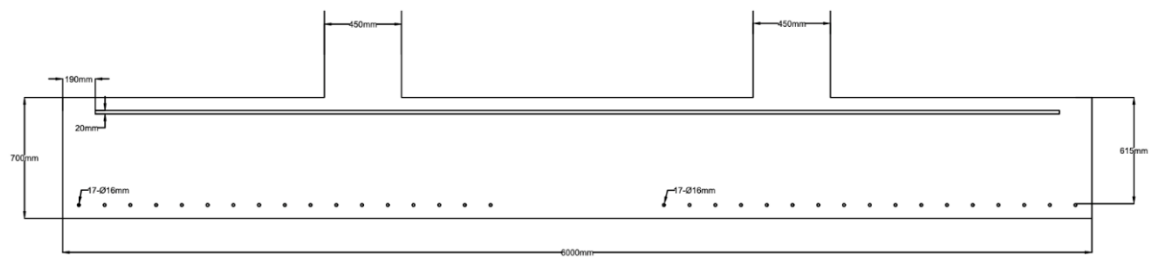


Figure 6.4.2. Longitudinal Cross-section View of Final Design of Combined Footing

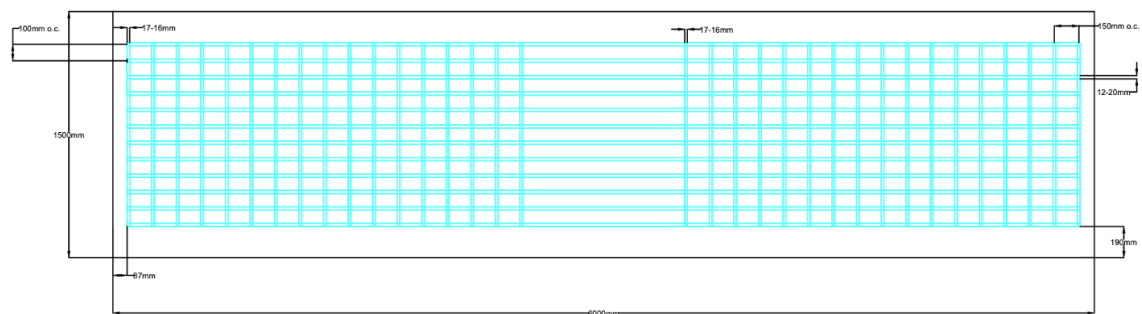


Figure 6.4.3. Top View of Final Design of Combined Footing

7 Cost Estimate

7.1 Unit Cost of Materials

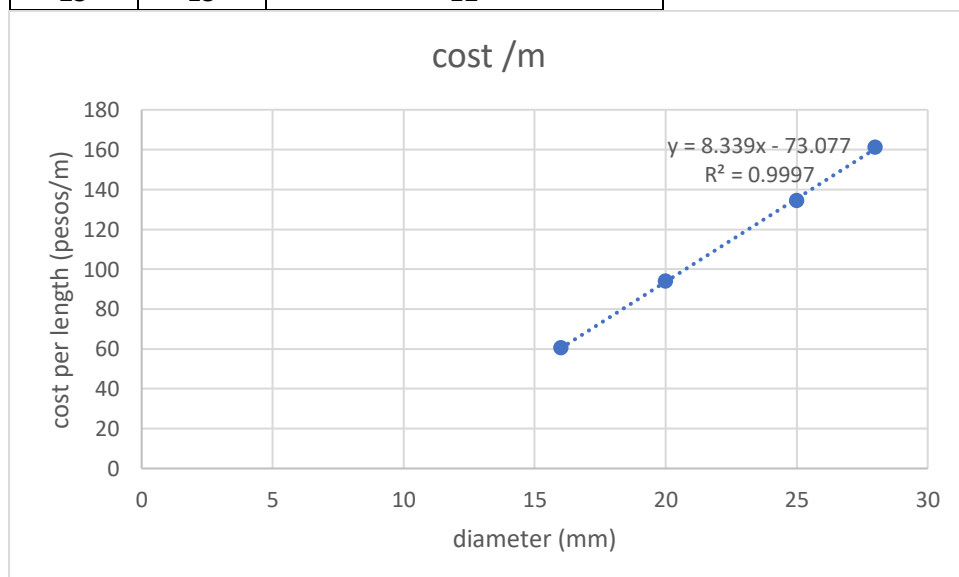
To estimate the cost of concrete, the concrete mix proportion given in Table 7.1.1 is utilized for the ratio between cement, sand, and gravel. From Table 7.1.2 and Figure 7.1.1, using correlation, we would be able to find the formula $n_{40} = 0.6f'_c$ wherein, n_{40} is the amount of 40 kg bag of cement needed on a $1m^3$ of concrete.

Table 7.1.1 Concrete Proportion

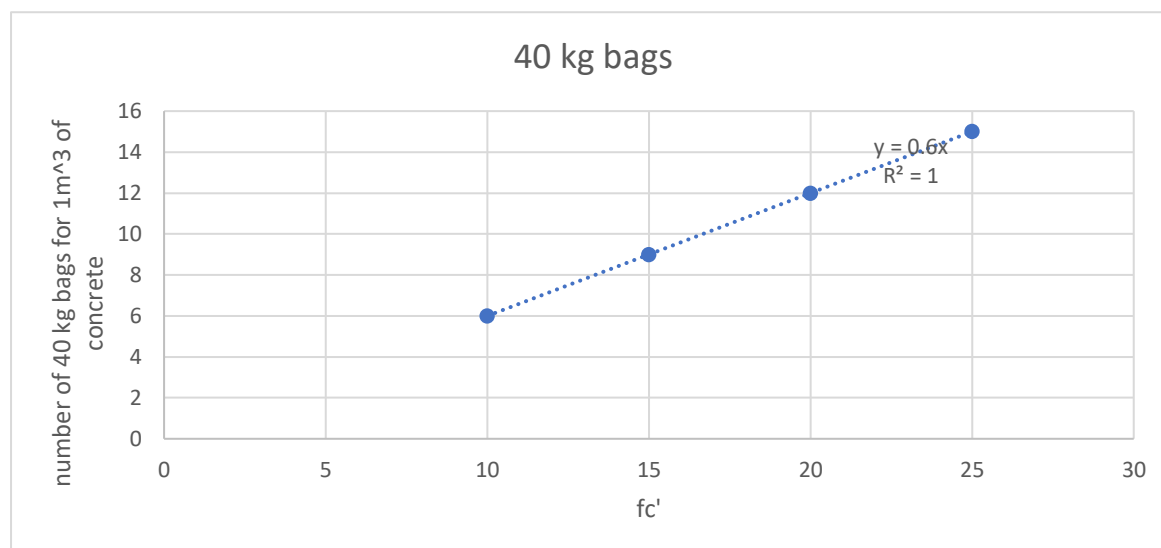
Mixture Class	Proportion	Cement in Bag		Sand cu.m	Gravel cu.m
		40 kg	50 kg		
AA	1:1.5:3	12.0	9.5	0.50	1.0
A	1:2:4	9.0	7.0	0.50	1.0
B	1:2.5:5	7.5	6.0	0.50	1.0
C	1:3:6	6.0	5.0	0.50	1.0

Table 7.1.2 Correlation between f'_c and the 40 kg bags.

f'_c	40 kg bags	50 kg bags (rounded to 0.5)
20	12	9.5
15	9	7
10	6	5
25	15	12



Note that the value for $f'_c = 25 \text{ MPa}$ is simply a linear extrapolation.

Figure 7.1.1. Linear Regression of f_c' and 40kg cement bags

The unit cost of materials is summarized in Table 7.1.3. The total cost of concrete per cubic meter can be estimated by utilizing the mix proportion for cement, gravel, and sand, and obtaining the correlation between the concrete strengths and available Portland cement (40kg) bags. Through linear regression, it was derived that the amount of 40kg bags needed for the concrete strength of 22.61MPa per cubic meter of concrete is 13.566.

Table 7.1.x Summary of Unit Cost of Concrete and Steel

Unit Cost of Materials		
Concrete		
f_c'	22.61	MPa
40kg bag per cu.m.	13.566	bags/m ³
Portland cement	259	PHP/bag
Cement cost per cu.m.	3513.594	PHP/m ³
Sand:cement ratio	0.5	
Gravel:cement ratio	1	
Sand cost per cu.m.	1435	PHP/m ³
Gravel cost per cu.m.	1435	PHP/m ³
Total cost of concrete per cu.m.	5666.094	PHP/m ³
Steel		
Rebar cost per kg	60	PHP/kg
Commercially available length	7.5	m
	9	m

The cost of reinforcing steel bars can be estimated through converting the required rebars with its corresponding weight since commercially available steel bars are sold by the

kilo. Table 7.1.4 shows the weight of rebar in kilogram per meter for the respective commercially available bar diameters. The commercial lengths that will be considered in this cost estimation will be 7.5 and 9.0 meters to cost optimization and reduction of wastage.

Table 7.1.4. Weight of Rebar for each given diameter

Weight of Rebar	
Diameter	Weight (kg/m)
10	0.168
12	0.89
16	1.58
20	2.465
25	3.851
28	4.831

7.2 Typical Exterior Square Footing

The concrete cost is estimated by obtaining the net volume of concrete by subtracting the total volume of steel from the gross volume of concrete. This is then multiplied by the unit cost of concrete per cubic meter, which is PHP 5,666.094 which results to a total of PHP 4,443.64 for the concrete cost of each exterior square footing.

TYPICAL (EXTERNAL) SQUARE FOOTING		
Number of footings	12	
Dimensions		
Base (B)	1500	mm
Length (L)	1500	mm
Thickness (T)	350	mm
Concrete		
Gross Volume of Concrete	787500000	mm3
Volume of Steel	3249160.786	mm3
Net Volume of Concrete	784250839.2	mm3
Volume in cu.m.	0.784250839	m3
Cost per cu.m.	5666.094	PHP/m3
Total Concrete Cost per footing	4443.638975	PHP
Steel		
Bar Diameter	16	mm3
Number of bars	5	pcs
<u>Along B</u>		
Rebar area (Ast)	201.0619298	mm2
Total rebar area along B	1005.309649	mm2
Concrete cover	92	mm
Rebar length	1316	mm
Total rebar length along B	6580	mm
Total rebar volume along B	1322987.498	mm3
	0.001322987	m3
<u>Along L</u>		
Rebar area (Ast)	201.0619298	mm2
Total rebar area along B	80	mm2
Concrete cover	92	mm
Rebar length	1316	mm
Total rebar length along B	6580	mm
Total rebar volume along B	1322987.498	mm3
	0.001322987	m3
<u>Dowels</u>		
Column length	450	mm
Column area (Ag)	202500	mm2

As,min	1012.5	mm ²
Rebar area (Ast)	201.0619298	mm ²
Number of rebars in section	6	pcs
Used rebar area (Ast)	1206.371579	mm ²
Development length	500	mm
Total length	3000	mm
Total volume of dowels	603185.7895	mm ³
	0.000603186	m ³
Total Rebar Estimate		
Total length required	16160	mm
	16.16	m
Commercially available length	9	m
Pieces needed (rounded up)	2	pcs
Total length	18	m
Wastage	1.84	m
Weight of bar per m	1.58	kg/m
Total rebar weight	28.44	kg
Unit cost per kg	60	PHP/kg
Total Rebar Cost per footing	1706.4	PHP
Cost Summary		
Concrete	4,443.64	PHP
Reinforcing bars	1,706.40	PHP
Cost per footing	6,150.04	PHP
Total cost of square footings	73,800.47	PHP

7.3 Isolated Square Footing

ISOLATED SQUARE FOOTING		
Number of footings	1	
Dimensions		
Base (B)	1750	mm
Length (L)	1750	mm
Thickness (T)	400	mm
Concrete		
Gross Volume of Concrete	1225000000	mm3
Volume of Steel	5560618.997	mm3
Net Volume of Concrete	1219439381	mm3
Volume in cu.m.	1.219439381	m3
Cost per cu.m.	5666.094	PHP/m3
Total Concrete Cost per footing	6909.45816	PHP
Steel		

Bar Diameter	20	mm
Number of bars	5	pcs
<u>Along B</u>		
Rebar area (Ast)	314.1592654	mm ²
Total rebar area along B	1570.796327	mm ²
Concrete cover	115	mm
Rebar length	1520	mm
Total rebar length along B	7600	mm
Total rebar volume along B	2387610.417	mm ³
	0.00238761	m ³
<u>Along L</u>		
Rebar area (Ast)	314.1592654	mm ²
Total rebar area along B	100	mm ²
Concrete cover	115	mm
Rebar length	1520	mm
Total rebar length along B	7600	mm
Total rebar volume along B	2387610.417	mm ³
	0.00238761	m ³
<u>Dowels</u>		
Column length	450	mm
Column area (Ag)	202500	mm ²
As,min	1012.5	mm ²
Rebar area (Ast)	314.1592654	mm ²
Number of rebars in section	4	pcs
Used rebar area (Ast)	1256.637061	mm ²
Development length	625	mm
Total length	2500	mm
Total volume of dowels	785398.1634	mm ³
	0.000785398	m ³
<u>Total Rebar Estimate</u>		
Total length required	17700	mm
	17.7	m
Commercially available length	9	m
Pieces needed (rounded up)	2	pcs
Total length	18	m
Wastage	0.3	m
Weight of bar per m	2.465	kg/m
Total rebar weight	44.37	kg
Unit cost per kg	60	PHP/kg
Total Rebar Cost per footing	2662.2	PHP
Cost Summary		

Concrete	6,909.46	PHP
Reinforcing bars	2,662.20	PHP
Cost per footing	9,571.66	PHP
Total cost of square footings	9,571.66	PHP

7.4 Rectangular Footing

RECTANGULAR FOOTING		
Number of footings	4	
Dimensions		
Base (B)	1250	mm
Length (L)	1500	mm
Thickness (T)	300	mm
Concrete		
Gross Volume of Concrete	562500000	mm3
Volume of Steel	2392756.345	mm3
Net Volume of Concrete	560107243.7	mm3
Volume in cu.m.	0.560107244	m3
Cost per cu.m.	5666.094	PHP/m3
Total Concrete Cost per footing	3173.620293	PHP
Steel		
<u>Along B</u>		
Bar Diameter	10	mm3
Number of bars	11	pcs
Rebar area (Ast)	78.53981634	mm2
Total rebar area along B	863.9379797	mm2
Concrete cover	120	mm
Rebar length	1010	mm
Total rebar length along B	11110	mm
Total rebar volume along B	872577.3595	mm3
	0.000872577	m3
<u>Along L</u>		
Bar Diameter	12	mm3
Number of bars	9	pcs
Rebar area (Ast)	113.0973355	mm2
Total rebar area along B	1017.87602	mm2
Concrete cover	119	mm
Rebar length	1262	mm
Total rebar length along B	11358	mm
Total rebar volume along B	1284559.537	mm3
	0.00128456	m3

<u>Dowels</u>		
Column length	450	mm
Column area (Ag)	202500	mm ²
As,min	1012.5	mm ²
Rebar area (Ast)	78.53981634	mm ²
Number of rebars in section	6	pcs
Used rebar area (Ast)	1206.371579	mm ²
Development length	500	mm
Total length	3000	mm
Total volume of dowels	235619.449	mm ³
	0.000235619	m ³
<u>Total Rebar Estimate</u>		
Total length required	25468	mm
	25.468	m
Commercially available length	9	m
Pieces needed (rounded up)	3	pcs
Total length	27	m
Wastage	1.532	m
Weight of bar per m	1.58	kg/m
Total rebar weight	42.66	kg
Unit cost per kg	60	PHP/kg
Total Rebar Cost per footing	2559.6	PHP
Cost Summary		
Concrete	3,173.62	PHP
Reinforcing bars	2,559.60	PHP
Cost per footing	5,733.22	PHP
Total cost of rectangular footings	22,932.88	PHP

7.5 Combined Footing

COMBINED FOOTING		
Number of footings	7	
Dimensions		
Base (B)	1500	mm
Length (L)	6000	mm
Thickness (T)	700	mm
Concrete		
Gross Volume of Concrete	6300000000	mm3
Volume of Steel	25078353.98	mm3
Net Volume of Concrete	6274921646	mm3
Volume in cu.m.	6.274921646	m3

Cost per cu.m.	5666.094	PHP/m3
Total Concrete Cost per footing	35554.29589	PHP
Steel		
<u>Along B</u>		
Bar Diameter	20	mm3
Number of bars	12	pcs
Rebar area (Ast)	314.1592654	mm2
Total rebar area along B	3769.911184	mm2
Concrete cover	190	mm
Rebar length	1120	mm
Total rebar length along B	13440	mm
Total rebar volume along B	4222300.526	mm3
	0.004222301	m3
<u>Along L</u>		
Bar Diameter	16	mm3
Number of bars	17	pcs
Rebar area (Ast)	201.0619298	mm2
Total rebar area along B	3418.052807	mm2
Concrete cover	87	mm
Rebar length	5826	mm
Total rebar length along B	99042	mm
Total rebar volume along B	19913575.65	mm3
	0.019913576	m3
<u>Dowels</u>		
Column length	450	mm
Column area (Ag)	202500	mm2
As,min	1012.5	mm2
Rebar area (Ast)	314.1592654	mm2
Number of rebars in section	6	pcs
Used rebar area (Ast)	1884.955592	mm2
Development length	500	mm
Total length	3000	mm
Total volume of dowels	942477.7961	mm3
	0.000942478	m3
<u>Total Rebar Estimate</u>		
Total length required	118482	mm
	118.482	m
Commercially available length	7.5	m
Pieces needed (rounded up)	16	pcs
Total length	120	m
Wastage	1.518	m

Weight of bar per m	2.465	kg/m
Total rebar weight	295.8	kg
Unit cost per kg	60	PHP/kg
Total Rebar Cost per footing	17748	PHP
Cost Summary		
Concrete	35,554.30	PHP
Reinforcing bars	17,748.00	PHP
Cost per footing	53,302.30	PHP
Total cost of combined footings	373,116.07	PHP
COMBINED FOOTING		
Number of footings	7	
Dimensions		
Base (B)	1500	mm
Length (L)	6000	mm
Thickness (T)	700	mm
Concrete		
Gross Volume of Concrete	6300000000	mm ³
Volume of Steel	25078353.98	mm ³
Net Volume of Concrete	6274921646	mm ³
Volume in cu.m.	6.274921646	m ³
Cost per cu.m.	5666.094	PHP/m ³
Total Concrete Cost per footing	35554.29589	PHP
Steel		
<u>Along B</u>		
Bar Diameter	20	mm
Number of bars	12	pcs
Rebar area (Ast)	314.1592654	mm ²
Total rebar area along B	3769.911184	mm ²
Concrete cover	190	mm
Rebar length	1120	mm
Total rebar length along B	13440	mm
Total rebar volume along B	4222300.526	mm ³
	0.004222301	m ³
<u>Along L</u>		
Bar Diameter	16	mm
Number of bars	17	pcs
Rebar area (Ast)	201.0619298	mm ²
Total rebar area along B	3418.052807	mm ²
Concrete cover	87	mm
Rebar length	5826	mm
Total rebar length along B	99042	mm
Total rebar volume along B	19913575.65	mm ³
	0.019913576	m ³
<u>Dowels</u>		
Column length	450	mm
Column area (Ag)	202500	mm ²

As,min	1012.5	mm ²
Rebar area (Ast)	314.1592654	mm ²
Number of rebars in section	6	pcs
Used rebar area (Ast)	1884.955592	mm ²
Development length	500	mm
Total length	3000	mm
Total volume of dowels	942477.7961	mm ³
	0.000942478	m ³
Total Rebar Estimate		
Total length required	118482	mm
	118.482	m
Commercially available length	7.5	m
Pieces needed (rounded up)	16	pcs
Total length	120	m
Wastage	1.518	m
Weight of bar per m	2.465	kg/m
Total rebar weight	295.8	kg
Unit cost per kg	60	PHP/kg
Total Rebar Cost per footing	17748	PHP
Cost Summary		
Concrete	35,554.30	PHP
Reinforcing bars	17,748.00	PHP
Cost per footing	53,302.30	PHP
Total cost of combined footings	373,116.07	PHP

7.6 Summary of Footing Costs

SUMMARY OF FOOTING COSTS			
Footing Type	Qty.	Cost per footing (PHP)	Total Cost (PHP)
Typical (Exterior) Square Footing	12	6,150.04	73,800.47
Isolated Square Footing	1	9,571.66	9,571.66
Rectangular Footing	4	5,733.22	22,932.88
Combined Footing	7	53,302.30	373,116.07
COST OF ALL FOOTINGS			479,421.08

7.7 Labor Cost Estimate

Using the table below:

Table 7.7.1. Footings work productivity rate (info.projectestimate, 2020).

#	ACTIVITY / DESCRIPTION	SIZE / DIMENSION	REMARKS	UNIT	TEAM COMPONENTS		AVE. OUTPUT - HOURS / UNIT		
							LABOR	EQUIPMENT	UNIT
1	Spread Footings	≤ 0.75m3	Incl. forms, rebar, & finish	Cubic Meters	1	Foreman	5.88	1.61	Hr / Cubic Meters
					5	Carpenter & Steelman			
					4	Laborer			
					0.25	Mason			
					0.25	Equip. Opt.			
					0.375	Concrete Vibrator			
					0.125	Concrete Pump			
2	Spread Footings	≥ 4.0m3	Incl. forms & rebar	Cubic Meters	0.25	Crane	4.9	1.34	Hr / Cubic Meters
					0.125	Truck			
					2	Power Tools			
3	Mat Foundation	≥ 1000m3	Incl. forms & rebar	Cubic Meters	0.5	Foreman	4.2	0.99	Hr / Cubic Meters
4	Mat Foundation	≥ 25 m3 to ≤ 500m3	Incl. forms & rebar	Cubic Meters	7	Carpenter & Steelman	5.15	1.21	Hr / Cubic Meters
5	Mat Foundation	≤ 25m3	Incl. forms & rebar	Cubic Meters	0.25	Mason	5.72	1.34	Hr / Cubic Meters
					4	Laborer			
					0.5	Equip. Opt.			
					0.25	Concrete Vibrator			
					0.25	Crane			
					0.25	Truck			
					0.125	Concrete Pump			
					2	Power Tools			

The team composition that we would assume is as follows:

Table 7.7.2. Team Composition

Foreman	1
Carpenter & Steelman	5
Laborer	4
Mason	0.25
Equipment Operator	0.25

Using the NCR Labor rates guide:

Table 7.7.3. NCR Labor Rates Guide – December of 2019

Description	Rate/Hr (Php)	Daily Rate (Php)
Foreman	133.50	1,068.00
Leadman	122.50	980.00
Heavy Equipment Operator	110.13	881.00
Highly Skilled Operator	110.13	881.00
Light Skilled Operator	101.50	812.00
Driver	92.88	743.00
Skilled Laborer	92.88	743.00
Light-skilled Laborer	87.63	701.00
Unskilled Laborer	75.88	607.00

Hence, we could assign as follows:

Table 7.7.4. Laborer Assignment

	Number needed	Assigned Category	Rate/Hr (Php)
Foreman	1	Foreman	133.5
Carpenter & Steelman	5	Skilled Laborer	92.88
Laborer	4	Unskilled Laborer	75.88
Mason	0.25	Skilled Laborer	92.88
Equipment Operator	0.25	Heavy Equipment Operator	110.13

The computation for the total gross area over all the footings would be as follows:

Table 7.7.5. Total Gross Area

Footing type	Number of columns covered	Length of Footing (mm)	Width of Footing (mm)	Thickness of Footing (mm)	Gross Volume per footing (mm ³)	Gross Volume per footing (m ³)	Columns covered for each footing	Total Gross Volume (m ³)
Exterior Square	12	1500	1500	350	7.88E+08	0.7875	1	9.45
Isolated Square	1	1750	1750	400	1.23E+09	1.225	1	1.225
Corner (Rectangular)	4	1500	1250	300	5.63E+08	0.5625	1	2.25
Interior (Combined)	14	6000	1500	300	2.70E+09	2.7	2	18.9
Sum								31.825

The calculation for the hours worked would be as follows:

Table 7.7.6. Days and Hours Needed

Name/Symbol	Value	Units
Total Gross Area	31.825	m ³
Labor Unit	5.88	hrs/m ³
Hours to Complete	187.131	hrs
Snapped to 8 hours/day	192	hrs
Days needed using team	24	days

We would use 192 hours.

After this the cost could be taken that:

Table 7.7.7. Cost of Labor

Role in Team	Number needed	Assigned Category	Rate/Hr (Php)	Hrs. of Work	Total Cost (Php)
Foreman	1	Foreman	133.50	192	25632.00
Carpenter & Steelman	5	Skilled Laborer	92.88	192	89164.80
Laborer	4	Unskilled Laborer	75.88	192	58275.84
Mason	0.25	Skilled Laborer	92.88	192	4458.24
Equipment Operator	0.25	Heavy Equipment Operator	110.13	192	5286.24

sum					182817.12
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Therefore, the total cost of Labor is:

Table 6.5-1 Footing Cost from Materials Estimate

₱182,817.12

11 References

Codes and Standards:

- National Structural Code of the Philippines 2015